Fatigue Assessment of Riveted Railway Bridges

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Abstract

The purpose of this paper is to present fatigue-related results obtained from finite element analyses of a typical riveted railway bridge. The first part of the paper deals with past case studies related to the fatigue assessment of various riveted railway bridges. The second part of the paper presents the results obtained from the finite element analyses of the wrought-iron bridge under typical present-day and assumed historical train loadings. These results are in the form of fatigue damage and associated remaining life estimates of the riveted connections. By fatigue-ranking the connections on an S-N basis and under different detail classifications, the most fatigue-critical connections are identified as being the inner stringer-to-cross-girder connections assuming full rotational connection fixity. Dynamic amplification is shown to affect remaining fatigue life estimates considerably.

Keywords : Riveted wrought-iron connections, fatigue damage, historical load model, finite element analysis

1.0 Introduction

Evaluation of the remaining fatigue life of riveted railway bridges is attracting attention from rail authorities all over the world, owing to continuously increasing number of such bridges reaching the end of their theoretical fatigue lives. Furthermore, the fatigue behaviour

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of wrought-iron and older steels, which were chiefly used for the construction of these bridges, is not well known. These observations coupled with the lack of information on the loading history of these bridges raise questions about their fatigue performance. As a result, the development of a comprehensive fatigue assessment methodology for riveted railway bridges is needed.

In this paper, different case studies related to the fatigue assessment of existing riveted railway bridges are presented and compared with each other. Furthermore, a global analysis of a typical riveted railway bridge, under present-day and historical train loading, is carried out in order to obtain stress histories appropriate for fatigue assessment. The fatigue criticality of the various riveted connections of the bridge is investigated by ranking them with respect to their fatigue damage. The latter is calculated based on three different fatigue detail classifications and taking into account dynamic amplification.

2.0 Case Studies on the Fatigue Assessment of Riveted Railway Bridges

Over the years, a number of riveted railway bridges have been assessed with respect to their remaining fatigue life. These were designed and constructed before standardisation and widespread use of design codes. Often, however, the unique features associated with these structures do not facilitate knowledge transfer and in many cases the conclusions are structure-specific.

In general, riveted bridges can be divided into truss and plate girder bridges. Truss bridges are used for larger spans, usually over 30 m, whereas plate girder bridges are used for shorter spans (around 10-20 m). The fatigue assessment procedure is quite similar for all types of bridges, however, fatigue criticality of structural details varies from bridge to bridge.

2.1 Truss Bridges

The fatigue assessment of broadly similar riveted truss bridges, with spans ranging between 35 and 53 m and years of construction ranging between 1906 and 1911, were carried out by Garg et al (1982), Szeliski and Elkholy (1984) and DiBattista et al (1998).

Garg et al (1982) developed an analytical model assuming rotational springs at the connections between floor-beams and stringers and rigid connections between the other members. Vibrations caused by train vehicles and bridge-vehicle interaction were taken into account. Fatigue lives were calculated by using an equivalent stress range (root-mean-cube) and the AREA (American Railway Engineering Association, 1996) detail category D for the hangers, stringers and floor-beams and the AREA (1996) detail category A for the stringer-to-floor-beam connections. The latter were obtained in terms of the bending behaviour of the connection angles that were used for these connections. The fatigue lives, which were found to lie between 50 and 140 years for the critical members, were determined by considering only freight traffic.

The rivet holes at the midspan location of stringers and floor-beams and the hanger-to-floor-beam connections were considered as being the fatigue-critical locations in the bridge investigated by Szeliski and Elkholy (1984). In contrast to the previous case, and after verifying that rivet holes were of good quality and the rivets tight, the details were classified as AREA (1996) category C. Past traffic was estimated using historical records whereas an annual growth of 5% was assumed for future traffic. By using the root-mean-square method, remaining fatigue lives of the stringers and floor-beams were estimated to be 7 and 16 years, respectively. As a result of this study, the bridge was retrofitted by replacing critical rivets with high-strength bolts.

Dibattista et al (1998) and Adamson and Kulak (1995) focused their attention on the truss diagonals and the stringers of a 38.1 m truss bridge. Comparison of field measurements with a structural model of this bridge revealed that fully-fixed connections resulted in better representation of the stringer stresses whereas pinned connections resulted in better representation of the diagonal stresses. Similar to the case study of Garg et al (1982), the details were classified as AREA (1996) category D. By using the effective stress range concept and a traffic history that included three different types of trains, the remaining fatigue lives of the critical members were found to be over 50 years.

Various wrought-iron truss bridges, with years of construction between 1859 and 1894 and spans ranging from 34 to 70 m have been fatigue-assessed during the last decade (Brühwiler and Kunz, 1993; Brühwiler, 1995; Keller et al, 1995; Bhavnagri, 1995). Details such as the chords of the main girders, stringers, cross-girders and different connections formed between the individual members were identified as being fatigue-critical. These were classified in the first three references according to Eurocode 3 (1993) as being detail category 71. On the other hand, Bhavnagri (1995) classified the investigated critical connection as Class D according to BS 5400 (1980). Both Brühwiler and Kunz (1993) and Bhavnagri (1995) considered past traffic models obtained from historical records to calculate the fatigue damage. In contrast, Keller et al (1995) considered only a particular freight train, with the heaviest axle load ever experienced by the bridge being representative of past and future traffic.

The fatigue assessment of all bridge members was carried out by Brühwiler and Kunz (1993), both deterministically and probabilistically. It was concluded that the bridge could be used for the next 25 years without any restrictions. On the other hand, the present day fatigue damage estimated by Bhavnagri (1995) for the fatigue-critical details was found to be approximately 20%.

By performing train simulations on a simple model which represented the bridge as a single continuous girder, Keller et al (1995) estimated the maximum stress range produced by the passage of the heaviest freight train and found it to be marginally higher than the fatigue limit. The present day damage due to the past traffic was found to be negligible.

Brühwiler (1995) developed an analytical model for part of a 34 m lattice-truss bridge to assess the fatigue criticality of the cross-girders. The stress ranges obtained from the analytical model were much higher than the stress ranges obtained through field measurements under passenger and freights trains due to the fact that the composite effect of the deck was not taken into account. Since the actual stress ranges were found to be well below the fatigue limit of the detail, Brühwiler concluded that fatigue was not an issue for this bridge.

2.2 Plate Girder Bridges

Several plate girder, riveted, railway bridges were assessed for fatigue by Wagh and Abrahams (1989), Philbrick et al (1995) and Tobias and Foutch (1997). The spans of these bridges ranged between 12 and 28 m and the construction years were between 1904 and 1917.

Midspan locations and flange cover plate terminations were judged to be the most critical details in the bridge investigated by Wagh and Abrahams. These were classified as AREA (1996) category D details. The traffic history was estimated through historical records for specific points in time, which were then used to represent the traffic for the corresponding periods between these times. The remaining fatigue life of the critical details, which was estimated by using an effective stress range coupled with Miner's Rule, was found to be in excess of 50 years.

Similar riveted, short-span, open-deck, plate girder bridges were assessed by Philbrick et al (1995) and Tobias and Foutch (1997). The latter investigation was carried out using a reliability procedure. By comparing field measurements with analytical results, Philbrick et al (1995) concluded that a fixed connection assumption in such bridges leads to a better representation of the member stresses. Stress ranges were estimated to be below the fatigue limit for AREA (1996) category D and, therefore, the fatigue damage was found to be negligible. By contrast, Tobias and Foutch considered a combined and modified form of the AREA (1996) category C and D S-N curves. Fatigue strengths and loadings were described through probability distributions. Using the root-mean-cube stress and Monte Carlo simulation it was found that the probabilistic remaining fatigue lives were very sensitive to freight car loads and wagon spacings.

The remaining fatigue life of three truss and three plate girder riveted bridges was investigated by Weiwen and Mohammadi (1996) on both a deterministic and a probabilistic basis. Stress ranges were described via probability density functions based on field measurements. Chords, hangers and diagonal members of the trusses and the bottom flanges of the plate girders and stringers were considered as being fatigue-critical and were classified according to BS 5400 (1980) as Class D details. Half of the components that were investigated were found to have probabilistic remaining fatigue lives less than 25 years.

A number of field measurements were conducted by Åkesson (1994) on various truss and girder riveted railway bridges with spans ranging from 10 to 104 m and constructed between 1903 and 1928. The maximum stress range for the bridges was found to be approximately 42 MPa. With the use of past historical train records and an assumed 5% annual growth in future traffic, remaining fatigue lives were estimated as being approximately equal to 30 years for four of the bridges and in excess of 50 years for the remaining bridges.

2.3 General Remarks

It can be seen from the case studies presented in the previous sections that, notwithstanding the different structural forms of old riveted railway bridges, a variety of assumptions are made for different bridge structures, either from the loading or the resistance point of view. However, the fatigue assessment procedures used for different bridges, which can be either deterministic or probabilistic, are generally similar. In the majority of the cases, the riveted details have been classified according to the AREA (1996) category D, BS 5400 Class D or the Eurocode 3 (1993) detail category 71. Furthermore, availability of historical records has led to the development of more detailed traffic histories thus allowing a more accurate evaluation of past fatigue damage. In some cases, field measurements have also established the validity of an analytical or structural model of the bridge in question. As expected, a large variation in the estimated fatigue lives of fatigue-critical details of these bridges is evident, ranging from few to well over one hundred years.

It should be noted that, in all these case studies, fatigue assessment was carried out by considering the primary stresses alone and it can be seen that, in general, these bridges possessed considerably reserve fatigue strength. This has been confirmed, through experiments and field measurements, by observing that the primary members are, generally, not fatigue-critical due to the low level of stress ranges that are experienced during the lifetime of the bridge (Fisher et al, 1984; Åkesson, 1994). Due to the inherent limitations of the analytical models of the riveted bridges under investigation, the effect of secondary stresses was not captured. However, secondary stress effects in riveted connections between the primary members of bridges were found to be one of the main reasons for fatigue damage (Fisher, 1984; Al-Emrani, 2005). The unavoidable rotational fixity of riveted connections and the variation in the clamping force of rivets have been identified as being the major causes leading to fatigue cracking in riveted connections (Al-Emrani, 2002; 2005).

3.0 Case Study of a Typical Riveted Railway Bridge

3.1 General Considerations

In this section, a UK-typical riveted railway bridge is considered for fatigue assessment. The finite element (FE) model of the bridge, which is representative of a large number of short-span, riveted, railway bridges around Europe and the UK, together with its relevant dimensions is shown in Fig. 1. The bridge, which is assumed to have been constructed circa 1900 and is simply supported on the three main girders, is modelled using 8-noded shell elements. A Young's modulus of 200 GPa, reported by Moy et al (2004) for wrought-iron material, and a Poisson's ratio of 0.3 are used for the FE analyses.

The riveted connections are assumed to be fully-fixed. Within the FE model this is achieved by tying all the members together at the locations of the connections. Previous investigations have revealed that full connection fixity results in lower midspan bending stresses at the stringers and cross-girders with an accompanying increase in the bending stresses near the connections (Adamson and Kulak, 1995; Philbrick et al, 1995). Furthermore, in these studies, the results obtained under the assumption of full connection fixity were found to be in better agreement with field measurements than the ones obtained under a pinned connections assumption (Adamson and Kulak, 1995). Parametric studies carried out by the authors have revealed that fully-fixed connections result in conservative remaining fatigue life estimates for the stringer-to-cross-girder connections (Imam et al, 2004), which, as will be shown later, are subject to higher fatigue damage than their cross-girder-to-main girder counterparts.

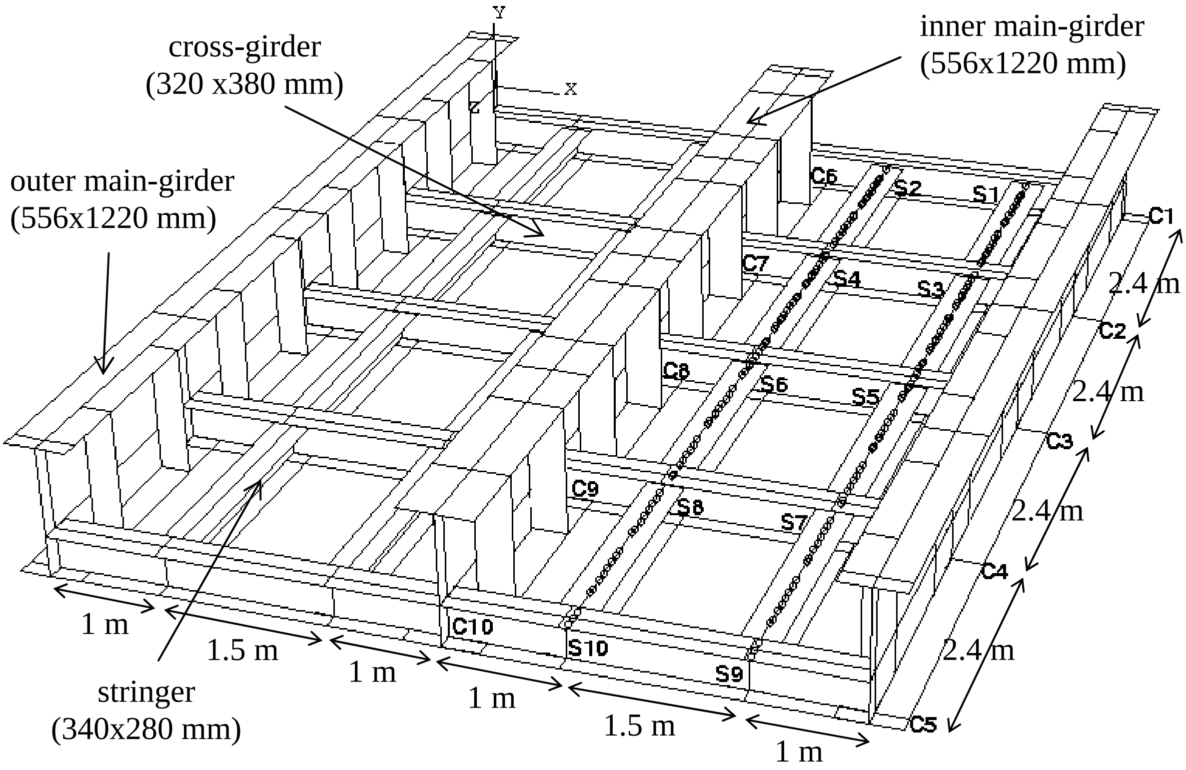

Figure 1 Bridge finite element model.

The bridge live loading is here represented by two railway traffic models. The BS 5400 (1980) medium traffic model is used to represent rail traffic from 1970 onwards. For the period from 1900 until 1970, a historical load model has been developed by the authors using historical train records (Imam et al, 2005).

For the purposes of the analyses, the trains are traversed in static steps of 1 m over one track of the bridge. Load spread due to rails or sleepers is not considered. The self-weight of the bridge elements and the superimposed dead load (sleepers and rails) are also included in the model.

Stress histories are obtained at distances of 250 mm and 300 mm from the stringer-to-cross-girder and the cross-girder-to-main girder interfaces, respectively. These distances were found to be the far-field locations where stress concentration effects due to the joining of the various members were found to diminish. Along the depth of the sections, stresses are reported at distances of 50 mm from the top or bottom flanges of the stringers and 60 mm for the cross-girders. These locations coincide with the position of the angle clip edges.

Next, the stress histories are converted into stress ranges by using the rainflow counting method (Downing and Socie, 1982) and the damage resulting from a single train passage is calculated using Miner's Rule (Miner, 1945). The riveted connections under investigation are classified either as Class B with the use of the appropriate stress concentration factor (2.4), hereinafter referred to as modified Class B, or Class D according to BS 5400 (1980). Class B can be assumed to represent the case of having a low or no clamping force in the rivets, whereas Class D is considered to be more representative of lapped or spliced connections with normal or high clamping force. A third detail classification, Class Wrought-Iron (WI), as proposed by the UK railway assessment code (Network Rail, 2001) for riveted, wrought-iron connections, is also considered. The S-N curves of these three detail classes are shown in Fig. 2. The change of slope from m to m+2, as proposed in BS 5400 (1980), occurs at 10^7 cycles corresponding to a stress range equal to the fatigue limit. In subsequent damage calculations, the BS 5400 mean minus two standard deviations, two-slope S-N curves are used for the modified Class B and Class D. Due to the simplified modelling of the riveted connections in the FE model, the fatigue damage of the individual elements of the connection (angles, rivets) is not investigated.

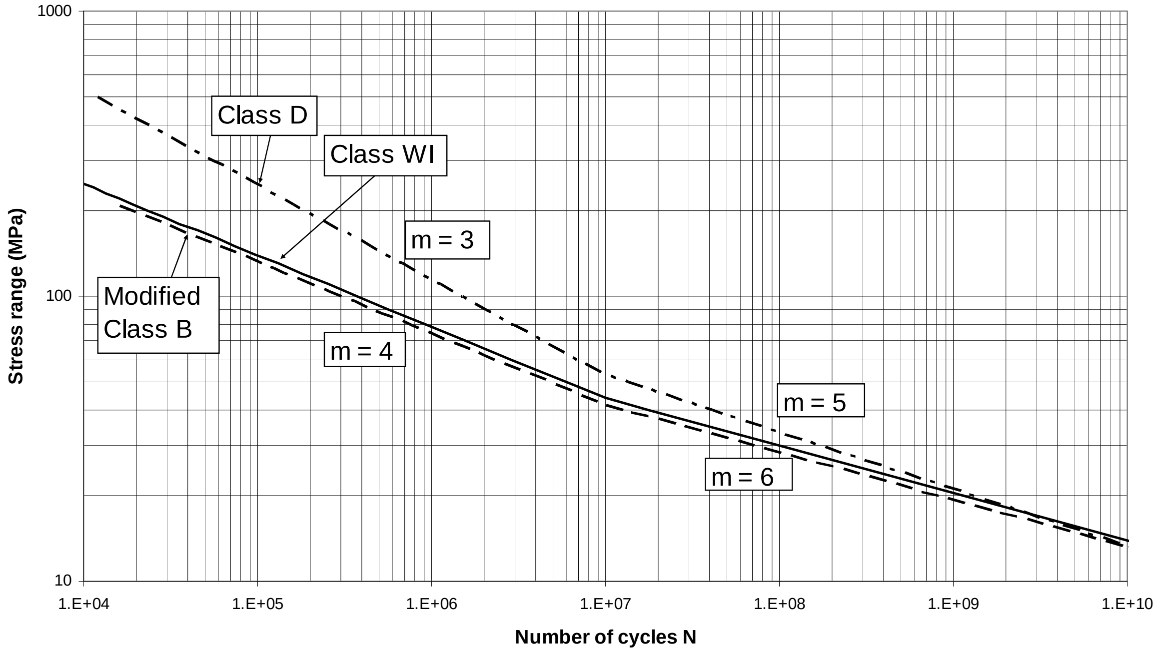

Figure 2 S-N curves for two BS 5400 (1980) fatigue classes and the WI class (Network Rail, 2001).

3.2 Results and Discussion

The numbering of the connections is shown in Fig. 1. In the following sections, the notation where the first symbol refers to the connection in question and the second one indicates the relevant direction is used. For example, S7-S5 refers to the connection at location S7 in the direction of connection S5. Results for the stringer-to-cross-girder (S) connections are reported near the bottom flange since stresses are found to be compressive near the top flange. By contrast, the stresses near the cross-girder-to-main girder (C) connections are found to be both tensile and compressive in the top flanges.

The histogram showing the total number of cycles versus the stress ranges experienced by connection S7-S5, assuming that the particular bridge was constructed in 1900, is shown in Fig. 3. This diagram was obtained by combining the historical (Imam et al, 2005) and the BS

5400 (1980) medium traffic load models. As can be seen in the figure, a very small number of stress ranges are above the fatigue limit for modified Class B and Class WI details, while for the Class D detail all the stress ranges lie below the corresponding fatigue limit. This is also found to be the case for most of the stringer-to-cross-girder connections.

Fig. 4 depicts the variation of the mean stress range level of connection S7-S5 where it is evident that there is a continuous increase in the level of the mean stress range that is experienced by this connection. The inset in the same figure shows that the Coefficient of Variation (CoV) for the historical load model (Imam et al, 2005) is higher than the COV associated with the BS 5400 (1980) medium traffic model. This may be due to the higher uncertainty in the assumed historical model which was roughly estimated by using historical records.

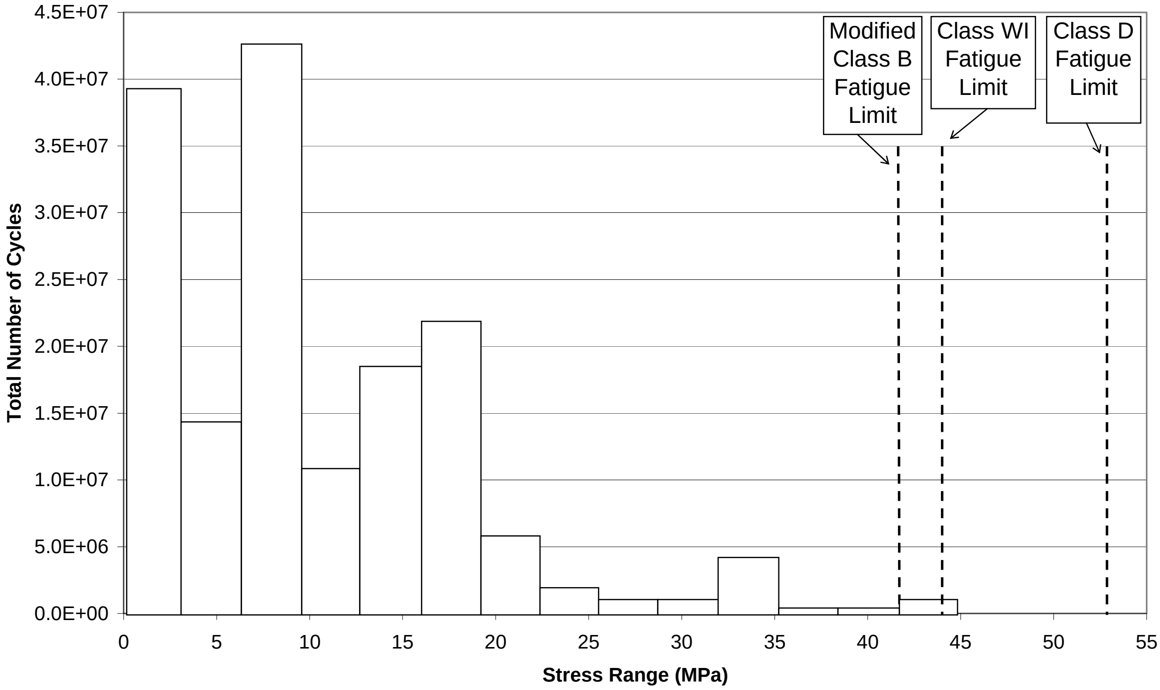

Figure 3 Stress range histogram for the stringer-to-cross-girder connection S7-S5 for the period 1900-2004.

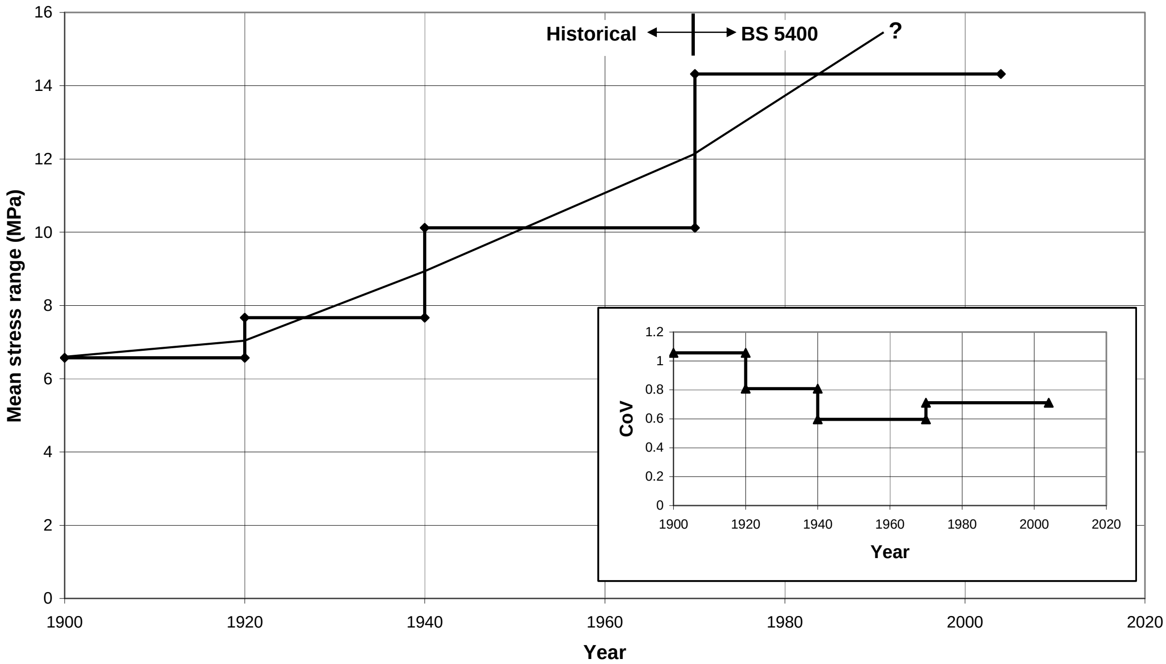

Figure 4 Evolution of mean stress range and CoV for connection S7-S5.

3.3 Total Damage of the Connections

The total damage of the stringer-to-cross-girder connections under the historical load model (Imam et al, 2005) and the BS 5400 (1980) medium traffic for the period 1900-2004 is shown in Fig. 5. Remaining fatigue life estimates are also indicated in the same figure. These estimates are calculated by extrapolating in the future present-day damage accumulation rates. Results are presented for all three assumed details (modified Class B, Class D and Class WI). Comparison of the results in Fig. 5 demonstrates that the damage of the inner stringer-to-cross-girder connections (S7-S5, S8-S6, S3-S5, S4-S6, see Fig. 1) is considerably higher than the outer stringer-to-cross-girder connections (S1, S2, S9 and S10, see Fig. 1). Damage ranking is not affected by the choice of the fatigue detail classification. In Fig. 6 the annual damage of the cross-girder-to-main-girder connections in the period 1970 to 2004 is compared with the corresponding annual damage of the stringer-to-cross-girder connections assuming modified Class B details. It can be seen that the stringer-to-cross-girder connections are far more prone to damage than their cross-girder-to-main-girder counterparts.

The evolution of damage for the two most highly-damaged connections (S7-S5 and S8-S6) is shown in Fig. 7 for all three detail classifications. In the same figure, results shown as “no low stress cycles” and obtained for the case of neglecting stress ranges which are below the cut-off stress range, as proposed by Network Rail (2001), are also included for connection S7-S5. The values of the cut-off stress range are 28.3, 30 and 33 MPa for modified Class B, Class WI and Class D, respectively. It can be seen that, irrespective of classification, after 1970 there is a considerable increase in damage accumulation due to the introduction of the heavier traffic. The assumption of a modified Class B results in the highest increase post-1970 due to the fact that the highest stress ranges lie above the corresponding fatigue limit as shown in Fig. 3. It can also be deduced from Fig. 7 that, for the case of modified Class B and Class WI details, approximately 30% of the S7-S5 connection fatigue strength has been expended over the last 34 years. Clearly, the introduction of heavier trains and possibly heavier axle loads in the future would result in even higher damage accumulation rates. Furthermore, by neglecting stress ranges below the cut-off value, there is an increase in the remaining fatigue life of connection S7-S5 from 80 years to 91 years and from 120 years to 134 years, for assumed modified Class B and Class WI, respectively. This increase is even higher for the case of an assumed Class D due to the higher slope of this class (m=5) in the second portion of the S-N curve as compared with the slope of the other classes (m=6).

3.4 Effect of Dynamic Amplification on Connection Damage

The stresses experienced by members and connections in short-span steel bridges can be considerably affected by dynamic amplification. Here, dynamic amplification factors (DAF)

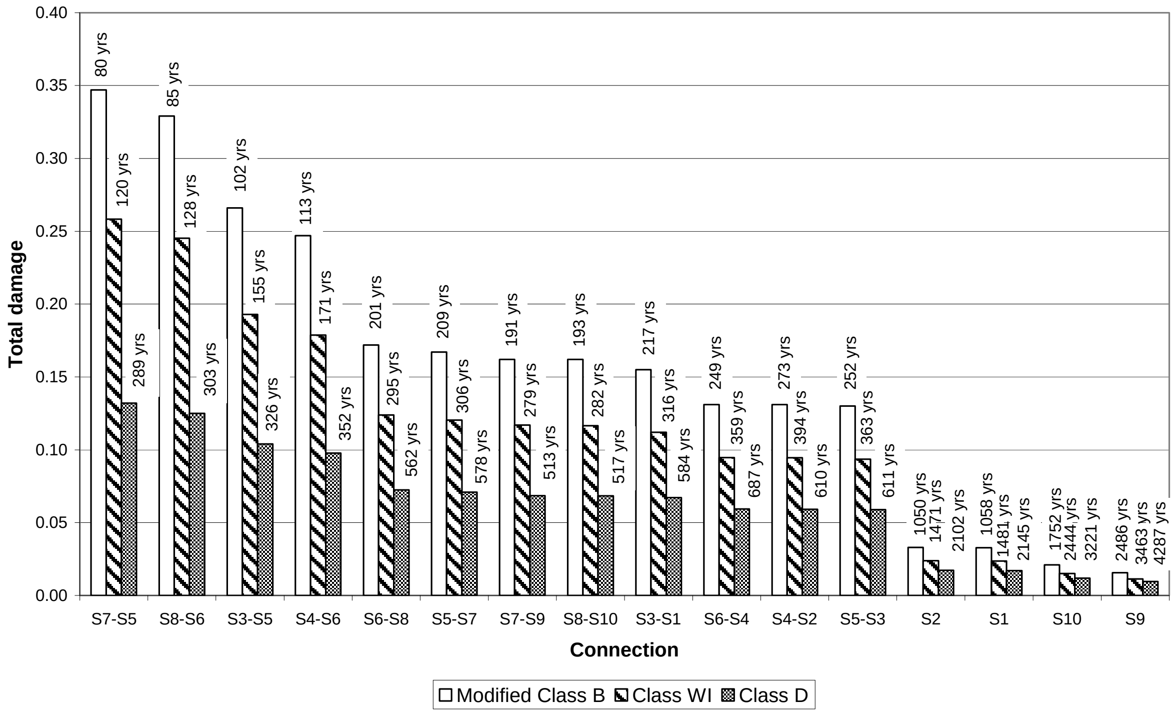

Figure 5 Stringer-to-cross-girder connection damage for the period 1900-2004 and remaining fatigue life estimates.

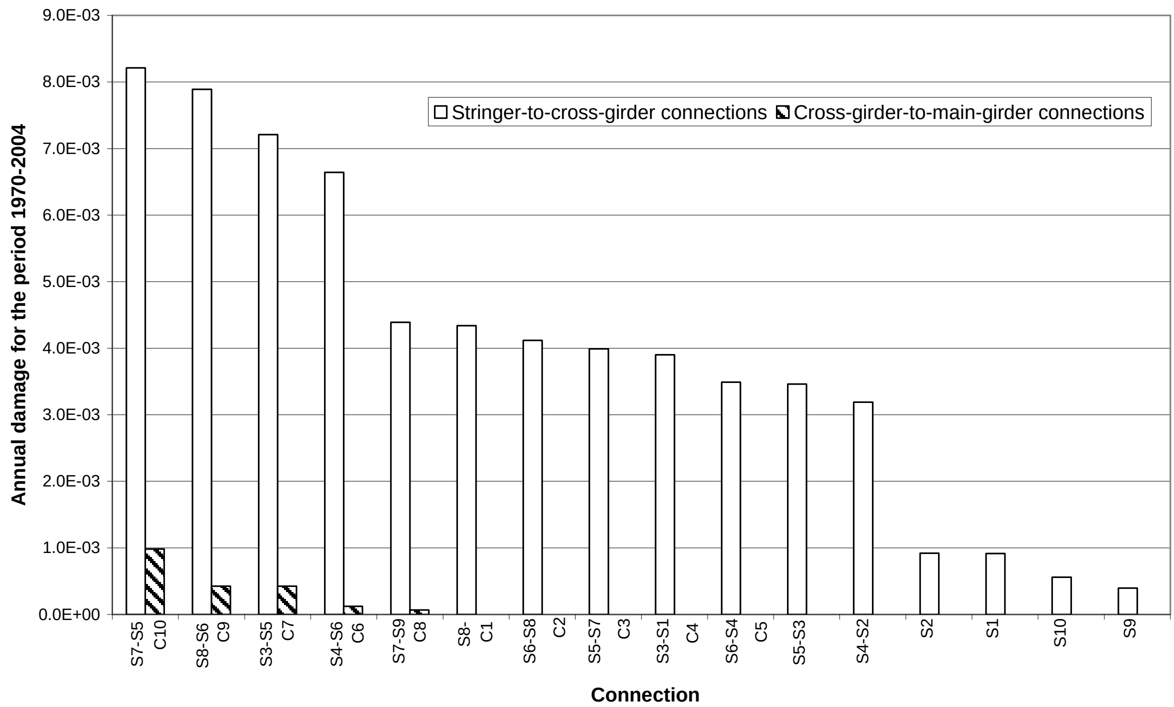

Figure 6 Comparison of cross-girder-to-main girder and stringer-to-cross-girder annual connection damage (modified Class B).

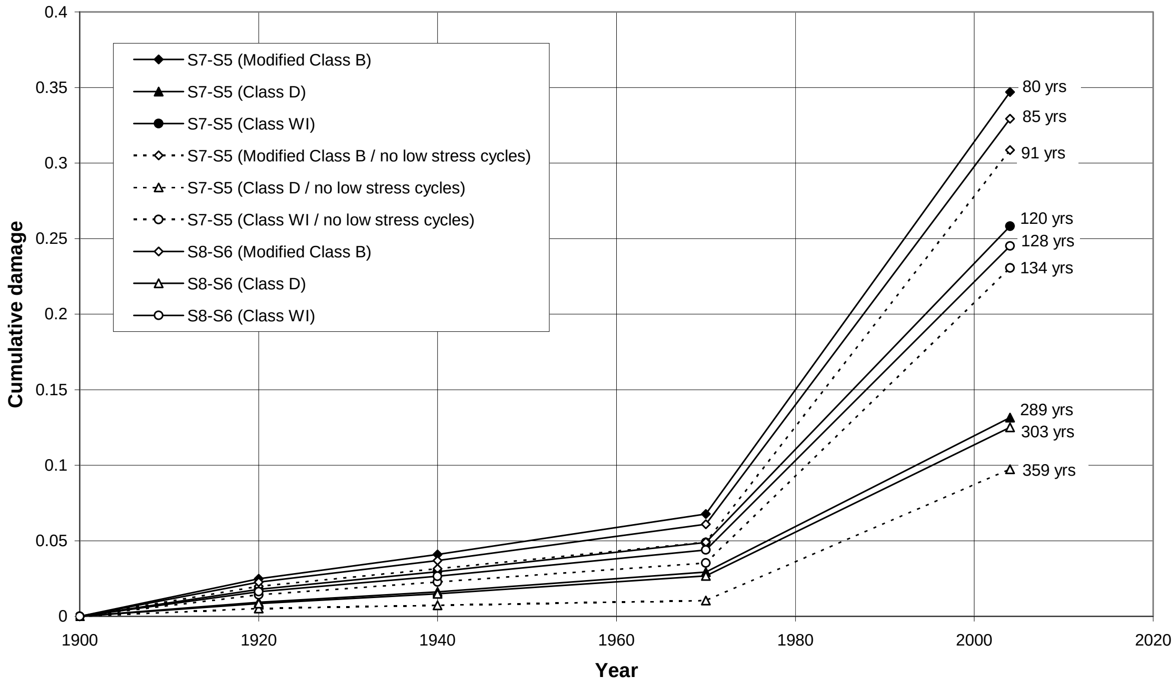

Figure 7 Cumulative damage vs time for connections S7-S5 and S8-S6.

obtained from various structural codes and published field measurements are applied to the statically calculated stresses.

The dynamic amplification factors are shown in Table 1 for different train speeds of the historical (Imam et al, 2005) and BS 5400 (1980) load models. The D23 factor is based on both theoretical studies and field measurements carried out by the European Rail Research Institute (D23, 1970). Dynamic amplification factors obtained from studies in the United States were derived from field measurements on short and medium span steel railway bridges (Byers, 1970; Tobias and Foutch, 1997) and are also shown in Table 1.

The effect of dynamic amplification on the cumulative damage as well as on the remaining fatigue life estimates for the most highly damaged connection S7-S5 is shown in Fig. 8 for modified Class B and Class D details and in the figure inset for Class WI. A considerable decrease in the remaining fatigue life of the connection due to the dynamic amplification can be seen in the figure. For the case of modified Class B connections, the remaining fatigue life is reduced from 80 years, which is the case without any dynamic amplification, to only 5-12 years depending on the DAF used. Similarly, assuming a WI Class for the connections (inset of Fig. 8) reduces the remaining fatigue life from 120 years to 21-31

	Train speed							
	BS 5400 trains (km/h)			Historical load model trains (mph)				
	72	80	160	30	40	50	60	70
Eurocode 1 (2003)	1.16	1.16	1.27	1.13	1.15	1.17	1.18	1.21
D23 (1970)	1.16	1.18	1.39	1.10	1.14	1.18	1.22	1.27
Byers (1970)	1.15	1.19	1.28	1.14	1.11	1.19	1.23	1.30
Tobias & Foutch (1997)	1.15	1.15	1.22	1.12	1.15	1.15	1.23	1.23
Network Rail (2001)	1.14	1.16	1.32	1.10	1.13	1.16	1.19	1.23

Table 1 Dynamic amplification factors reported from different sources for various train speeds.

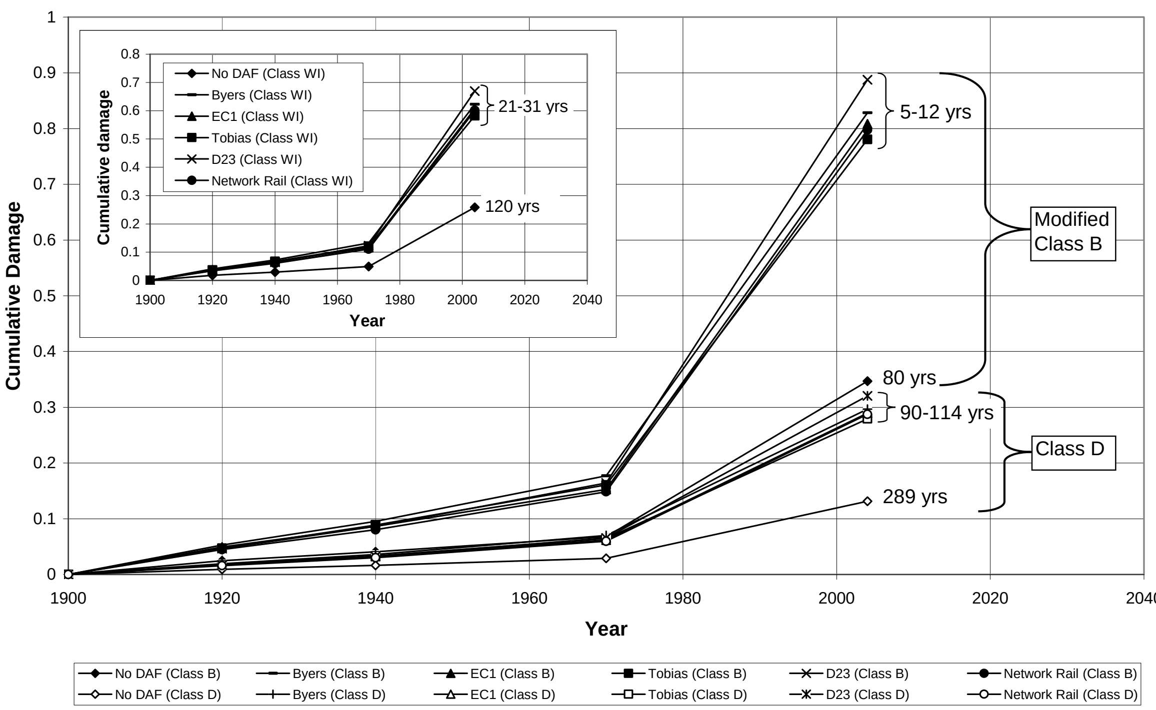

Figure 8 Cumulative damage of connection S7-S5 with respect to different dynamic amplification factors.

years. Finally, for the case of assumed Class D connections, the remaining fatigue life is found to decrease from 289 years (no DAF) to 90-114 years. The sensitivity of the remaining fatigue lives to the dynamic amplification can be attributed to the increase of some of the highest stress ranges, which were previously below the fatigue limit, to levels above the fatigue limit. The shift from the second (slope m+2) to the first (slope m) branch of the S-N curve and the power law nature of fatigue damage naturally result in these large differences. However, the assumption that all stress ranges are multiplied by the same dynamic amplification factor provides a simple approximation to an otherwise complicated phenomenon since, in general, the dynamic stress histories will be different from their static counterparts.

3.5 Effect of Train Traffic on Connection Damage

The results presented so far were obtained by loading one track of the bridge only. Since the investigated bridge has two tracks, the effect of trains passing over the opposite track on connection damage is also investigated. It is assumed that the train frequency is equal in both tracks and that the latter are not simultaneously loaded. Fig. 9 shows results accounting for the effect of the second track, for connection S7-S5, compared to the results of Fig. 7 which were obtained by loading only one track of the bridge. It can be seen that passage of trains over the second track leads to a slight decrease, by approximately 4-5 years, in the remaining fatigue life of the connection for all assumed detail classes. Therefore, it can be said that the damage of the connections on one loaded side of the bridge is not considerably affected by the passage of train traffic over the opposite track of the bridge.

The simultaneous passage of two trains over the bridge is also investigated here. Fig. 10 shows the damage calculated, using Miner's sum, under the simultaneous passage of two BS

5400 No 7 trains over both tracks of the bridge. Loading case 1 refers to the case where both trains enter the bridge at the same instance, whereas loading case 2 refers to the case where the two trains enter the bridge at different instances. The results are presented for the case of assumed modified Class B and Class D (see inset of figure) connections without considering any dynamic amplification. A considerable increase in the damage of the majority of the connections (by as much as 400%) due to two-train loading may be seen in the figure. The increase is found to be considerably higher in the cross-girder-to-main girder connections. However, despite this increase, their damage was still found to be less critical than the stringer-to-cross-girder connection damage reported in Fig. 10. It should be noted that the annual occurrence of two-train loading is rather low, therefore, its effect on the total damage of the connections can be regarded as negligible.

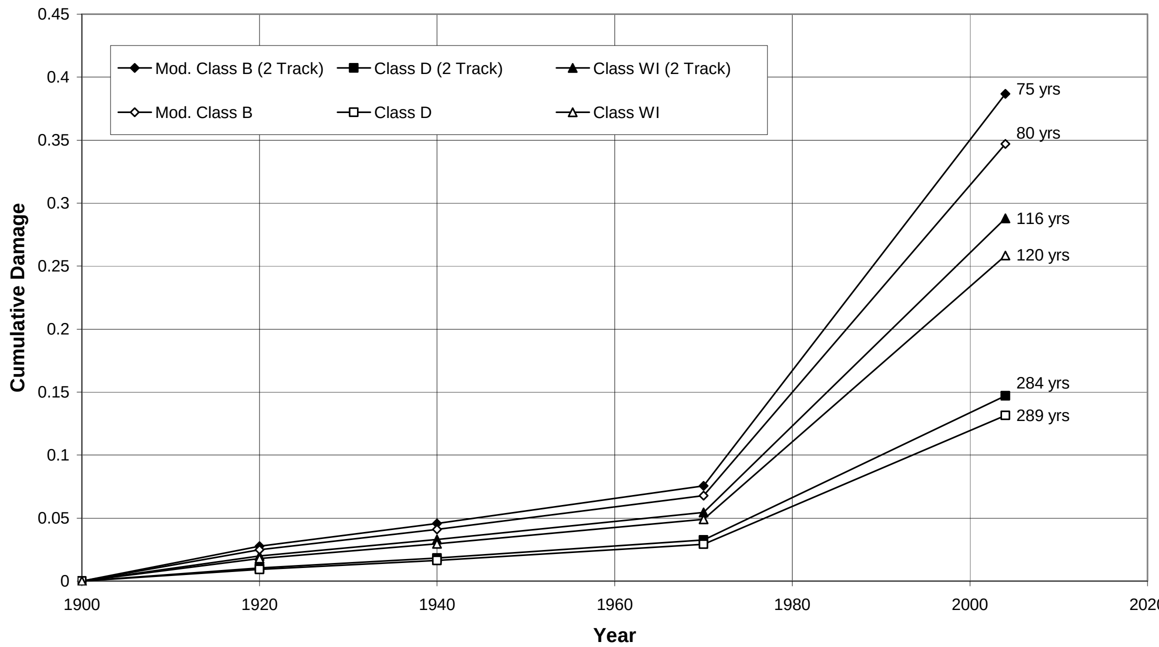

Figure 9 Effect of second track loading on cumulative damage of connection S7-S5 (no DAF).

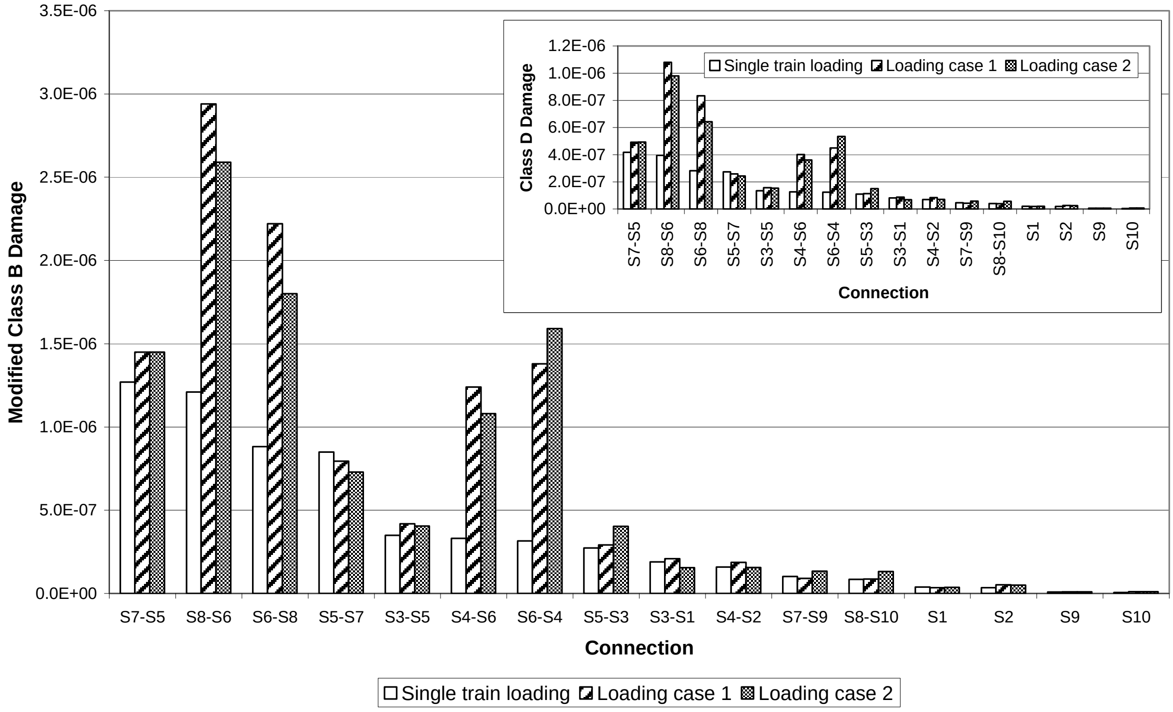

Figure 10 Stringer-to-cross-girder connection damage for different loading cases (no DAF).

5.0 Conclusions

In this paper, several case studies on the fatigue assessment of riveted railway bridges have been presented. Due to the large number of different structure types, the unique features associated with each of these and the various assumptions regarding structural behaviour in terms of loading and resistance, the development of a comprehensive fatigue assessment methodology for riveted railway bridges is not an easy task. As a first step towards this objective, finite element analyses of a typical riveted railway bridge were carried out in order to assess the fatigue criticality of its connections.

Overall, the inner stringer-to-cross-girder connections were identified as being the most fatigue-critical under the assumption of fully-fixed connections. Dynamic amplification and two-train simultaneous loading were found to result in a considerable increase in the connection damage. Fatigue damage was found to increase marginally when loading of both tracks was considered. The damage accumulation rate was found to be small in the pre-1970 period under a historical load model but showing a considerable increase with the introduction of the BS 5400 trains (post-1970). Overall, connections S7-S5 and S8-S6 were identified as being the most critical with regard to fatigue damage. A more detailed investigation of these connections using Fracture Mechanics principles is currently under way.

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References

ABAQUS. (2002). "*Standard User's Manual Version 6.3*", Hibbitt, Karlsson & Sorensen, Inc.

Adamson, D. E. and Kulak, G. L. (1995). "Fatigue Tests of Riveted Bridge Girders." *Structural Engineering Report No 210*, University of Alberta, Edmonton.

Åkesson, B. (1994). "Fatigue Life of Riveted Railway Bridges." *PhD Thesis. Publ. S 94:6,* Chalmers University of Technology, Göteborg.

Al-Emrani, M. (2002). "Fatigue in Riveted Railway Bridges – A Study of the Fatigue Performance of Riveted Stringers and Stringer-to-Floor-Beam Connections." *PhD Thesis,* Chalmers University of Technology, Göteborg.

Al-Emrani, M. (2005). "Fatigue Performance of Stringer-to-Floor-Beam Connections in Riveted Railway Bridges." *Journal of Bridge Engineering*, ASCE, 10(2), p. 179-185.

AREA. (1996). "*Manual for Railway Engineering*." American Railway Engineering Association, Washington D. C.

Bhavnagri, V. S. (1995). "Analysis and Testing of an Old Lattice Girder Rail Bridge." *Australian Civil Engineering Transactions*, CE37(3), p. 229-239.

Brühwiler, E. and Kunz, P. (1993). "Remaining Fatigue Life of a Riveted Railway Bridge." *Proc. IABSE Colloquium: Remaining Structural Capacity*, Copenhagen, p. 375-383.

Brühwiler, E. (1995). "Measurements for Evaluating the Remaining Service Life of a Riveted Bridge." *Proc. IABSE Symposium : Extending the Lifespan of Structures*, San Francisco, p. 1023-1028.

BS 5400. (1980). "*Steel, Concrete and Composite Bridges: Part 10 : Code of Practice for Fatigue*", British Standards Institute, London.

Byers, W. G. (1970). "Impact from Railway Loading on Steel Girder Spans." *Journal of the Structural Division*, ASCE, 96(ST6), p. 1093-1103.

D23 Specialists' Committee. (1970). "*RP15: Determination of Dynamic Forces in Bridges – Statistical Investigation of the Dynamic Effects Measured in Bridges.*" Technical Report, International Union of Railways UIC, Utrecht.

Dibattista, J. D., Adamson, D. E. J. and Kulak, G. L. (1998). "Evaluation of Remaining Fatigue Life for Riveted Truss Bridges." *Canadian Journal of Civil Engineering*, 25(4), p. 678-691.

Downing, S. D. and Socie, D. F. (1982). "Simple Rainflow Counting Algorithms." *International Journal of Fatigue*, 4(1), p. 31-40.

Eurocode 1. (2003). "*Actions on Structures: Part 2: Traffic Loads on Bridges.*" EN 1991-2.

Eurocode 3. (1993). "*Design of Steel Structures : Part 1.1 : General Rules and Rules for Buildings*." ENV 1993-1-1:1992.

Fisher, J. W. (1984). *Fatigue and Fracture in Steel Bridges: Case Studies*. John Wiley & Sons, New York.

Fisher, J. W., Yen, B. T. and Wang, D. (1984). *NCHRP Report No 302: Fatigue and Fracture Evaluation for Rating Riveted Bridges*. Transportation Research Board, National Research Council, Washington D.C.

Garg, V. K., Chu, K. H. and Wiriyachai, A. (1982). "Fatigue Life of Critical Members in a Railway Truss Bridge." *Earthquake Engineering and Structural Dynamics*, 10, p.779-795.

Imam, B., Righiniotis, T. D. and Chryssanthopoulos, M. K. (2004). "Connection Fixity Effects on Stress Histories in Riveted Rail Bridges." *Proc. 2nd International Conf. Bridge Maintenance, Safety and Management*, Kyoto, (CD-ROM).

Imam, B., Righiniotis, T. D. and Chryssanthopoulos, M. K. (2005). "Remaining Fatigue Life Estimates for Riveted Railway Bridges." *Proc. 5th International Conf. Bridge Management*, Thomas Telford, UK.

Keller, A., Brühwiler, E. and Hirt, M. A. (1995). "Assessment of a 135 Year Old Riveted Railway Bridge." *Proc. IABSE Symposium : Extending the Lifespan of Structures*, San Francisco, p. 1029-1034.

Miner, M. A. (1945). "Cumulative Damage in Fatigue." *Journal of Applied Mechanics*, 12, p. 159-164.

Moy, S. S. J., Clark, J. and Clarke, H. (2004). "The Strengthening of Wrought Iron Using Carbon Fibre Reinforced Polymer Composites." *Proc. Advanced Polymer Composites for Structural Applications in Construction (ACIC)*, Surrey, UK, p. 258-265.

Network Rail. (2001). "*RT/CE/C/025. Railtrack Line Code of Practice: The Structural Assessment of Underbridges*", Railtrack.

Philbrick, T. W., Zodo, G. W. and Schiff, S. D. (1995). “Fatigue Assessment of Through Plate Girder Railway Bridges.” *Journal of Structural Engineering*, ASCE,121(11), p.1613-1619.

Szeliski, Z. L. and Elkholy, I. A. (1984). “Fatigue Investigation of a Railway Truss Bridge.” *Canadian Journal of Civil Engineering*, 11, p. 625-631.

Tobias, D. H. and Foutch, D. A. (1997). “Reliability-Based Method for Fatigue Evaluation of Railway Bridges.” *Journal of Bridge Engineering*, ASCE, 2(2), p. 53-60.

Wagh, V. P. and Abrahams, M. J. (1989). “Fatigue Evaluation of a Riveted Railroad Bridge.” *Proc. Sessions Related to Structural Materials – Structures Congress,* San Francisco, p. 179-188.

Weiwen, L. and Mohammadi, J. (1996). “Assessment of Remaining Fatigue Life of Existing Railway Riveted Bridges.” *Probabilistic Mechanics and Structural Reliability*, ASCE, p. 42-45.